Simple Methods for Calculating the Structural Reliability for Different Knowledge Levels

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ABSTRACT

The recent European codes such as Euro Code 8 seem to synthesize the effect of structural modeling uncertainties in the so-called confidence factors (CF) that are applied to mean material property values. However, the effect of the application of the confidence factors on structural reliability is not explicitly stated. An alternative approach featured in the SAC-FEMA guidelines, considers the effect of both ground motion uncertainty and the structural modeling uncertainties on the global performance of the structure, in a closed-form analytical safety-checking format. This work employs an approximate semi-deterministic method to study the confidence factors from the point of view of the characterization of uncertainties and structural reliability assessment. Moreover, an efficient Bayesian method is presented that can estimate both the robust structural reliability and also the joint probability distributions for structural fragility parameters, based on a small sample of structural model realizations and ground motion records. Based on findings featured in this work, a set of perspectives for the future European codes are outlined.

1 INTRODUCTION

Many European countries are subject to a considerable risk of seismic activity. Quite a few of these countries enjoy a rich patrimony of existing buildings, which for the most part were built before the specific seismic design provisions made their way into the constructions codes. Therefore, the existing buildings can potentially pose serious fatality and economic risks in the event of a strong earthquake. One main feature distinguishing the assessment of existing buildings from that of the new construction is the large amount of uncertainty present in determining the structural modeling parameters. The recent European codes seem to provide a level of conservatism in the assessment of existing buildings, in the application of the (inverse of the larger than unity) *confidence factors* (*CF*) to mean material property values. These confidence factors are determined as a function of the *knowledge levels* (*KL*). The knowledge levels are determined based on the amount of tests and inspections performed on the existing building. Table 1 illustrates the three KL's, namely, limited, extended and comprehensive, based on the amount of in-situ tests and inspections performed.

Table 1. Recommended minimum requirements for different levels of inspection and testing (* for each type of primary element).

KL	Inspections of details (% elements*)	Testing of Materials (sample/floor)
Limited	20	1
Extended	50	2
Comprehensive	80	3

The application of the confidence factor seems to be a deterministic method for addressing an inherently probabilistic problem. With the emerging of probability-based concepts such as life-cycle cost analysis and performance-based design, the question arises as to what the CF would signify and would guarantee in terms of the structural seismic reliability [Jalayer et al. 2008, Franchin et al. 2008]. This would not be possible without a thorough characterization of the uncertainties in the structural modeling parameters [Monti and Alessandri 2008 and Jalayer et al. 2008]. Another issue regards the definition of the KL. The current code definition in Table 1 leaves a lot of room for interpretation; it is independent of the spatial configuration and the outcome of the test results. Moreover, the logical connection between the numerical values for the confidence factors and the onset of the knowledge levels is not clear. A simple approximate semi-deterministic method is employed in this work in order to calculate the structural reliability based on code recommendations.

An alternative probabilistic and performance-based approach is adopted in the American Department of Energy Guidelines DOE-1020 and in SAC-FEMA guidelines. This simplified approach leads to an analytic and closed-form solution which compares the factored demand against factored capacity. The factored demand and capacity are respectively equal to median demand and capacity multiplied by some factors. The magnifying demand factors and the de-magnifying capacity factors take into account all sources of uncertainty, such as record-to-record variability, structural modeling uncertainty and the uncertainty in the capacities. This approach that is recently known as the *Demand Capacity Factor Design* (DCFD) [Cornell et al., 2002] takes into account the overall effect of the various types of uncertainties on a global structural performance parameter. Therefore, in the case of existing buildings, there is a need for a method that can evaluate the global parameters reflecting the overall effect of structural modeling uncertainties.

The Bayesian framework for probabilistic inference seems to be a perfect basis for taking into account the results of tests and inspection in updating the structural model. The authors in a previous work [Jalayer et al. 2008] have demonstrated how the advanced simulation methods based on Bayesian updating can be used to both update the structural reliability and also the probability distribution for the modeling parameters, in the presence of test and inspection results. However, the application of the advanced simulation schemes requires a large number of structural analyses and there seems to be a need for less computationally intensive methods for updating the structural model and structural reliability. The authors [Jalayer et al., 2009] have employed an efficient Bayesian simulation-based method for robust estimation of structural reliability. This method exploits a relatively small number of structural analyses in order to yield the *robust reliability* for the structure in question. The term robust herein refers to the fact that the reliability is calculated taking into account all possible structural models and their relative plausibilities.

1.1 The Structural Performance Parameter

The structural performance parameter in the context of this work is a particular kind of demand to capacity ratio. This parameter which denoted as *Y*, assumes the value of unity on the verge of the limit state *LS*. In the case of static analyses, the capacity spectrum method [Fajfar, 1990] is used to obtain the global demand to capacity ratio. In the case of dynamic analyses, the *cut-set* concept in reliability theory is employed to find the critical component demand to capacity ratio that takes the structure closer to the onset of the limit state *LS*. This critical demand to capacity ratio corresponds to the strongest component of the weakest structural mechanism [Jalayer et al., 2007].

2 METHODOLOGY

In this section a methodology for taking into account the sources of modeling uncertainties in the probabilistic performance assessment of existing buildings is presented.

2.1 The SAC-FEMA Methodology

In the case of static analyses, the SAC-FEMA formulation reduces to the following:

$$\eta_Y \cdot e^{\frac{1}{2}\beta^2{}_Y} \leq 1 \qquad \text{Eq. 1}$$

Where η_y is the median and β_Y is the standard deviation of the logarithm for the probability distribution for the structural performance parameter *Y*. If *Y* is described by a Lognormal distribution, this is equivalent to checking whether the mean value for the structural performance parameter is less than unity. The parameter β_Y represents the overall effect of uncertainties on the probability distribution for the structural performance parameter. When record-to-record variability is considered, the formulation is modified as:

$$\eta_Y(P_o) \cdot e^{\frac{1}{2}\frac{k}{b}(\beta^2{}_{Y|S_a} + \beta^2{}_{UC})} \leq 1 \qquad \text{Eq. 2}$$

Where P_o is an acceptable threshold for structural failure probability and $\eta_Y(P_o)$ is the median structural performance parameter corresponding to the acceptable probability P_o. The terms $\beta_{Y|Sa}$ and β_{UC} represent the effect of record-to-record variability and structural modeling uncertainties, respectively, on the total dispersion in the structural performance parameter given spectral acceleration.

2.2 Characterization of the Uncertainties

It is assumed that the vector $\underline{\theta}$ represents all the uncertain parameters considered in the problem. The vector $\underline{\theta}$ can include the uncertainties in the mechanical properties of the materials, in the structural construction details (a.k.a., *defects*) and in the representation of the ground motion uncertainty. One of the main characteristics of the construction details is that possible deviations from the original configurations are mostly taken into account in those cases leading to undesirable effects. This justifies why the uncertainties related to construction details are usually referred to as the structural defects.

Three types of uncertainties are considered herein, namely, the uncertainty in the ground motion input, the uncertainty in the material mechanical properties, and the uncertainties in the structural detailing parameters. A set of 30 ground motion records are chosen from the European strong motion database for soil type B ($400 < V_s < 600$ m/s), with moment magnitude between 5.3 to 7.2 and the epicentral distance between 7 and 87 km. Moreover, a set of 7 ground motion records are chosen compatible with the spectrum of Euro Code 8. The parameters identifying the prior probability distributions for the material mechanical properties (concrete strength and the steel yielding force) have been based on the values typical of the post world-war II construction in Italy [Verderame et al. 2001a,b]. Table 2a shows these parameters that are used to define the Lognormal probability distributions for the material properties. The prior probability distributions for the structural detailing parameters are defined based on qualitative prior information coming from expert judgment or based on *ignorance* in the extreme case [Jalayer et al., 2008]. Table 2b shows the specifications used to construct the prior probability distributions for the structural detailing parameters. It shows a list of possible defects, their probability distribution and correlation characteristics.

Table 2. The uncertainties in the material properties (systematic per floor).

material	Type	Median	COV
f_c	LN	165	0.15
f_y	LN	3200	0.08

Table 2. The uncertainties in details (systematic).

defect	Type	Min	Max
spacing of shear rebar	Uniform (beams)	15cm	30cm
spacing of shear rebar	Uniform (column)	20cm	35cm

2.2.1 Updating the probability distributions

The probability distributions for the structural modeling parameters can be updated employing the Bayesian framework for inference. It is assumed that the material properties are homogeneous across each floor or construction zone. Therefore, the material property value assigned to each floor can be thought of as an average of the material property values across the floor/zone in question. The results of tests and inspections for each floor can be used to update the probability distribution for the mean material property across the floor. Figure 1 illustrates an example where the test results have verified the nominal value for different levels of knowledge. It can be observed that the updated curve has the same median but has its dispersion reduced.

Assuming that the probability not having a construction defect in a member is equal to *f*, the probability distribution for *f* can be updated using the test results. If the test results indicate that of *n* cases observed n_d of them demonstrate a defect, the probability distribution for *f* can be updated according to the Bayes formula:

$$p(f \mid D) = \frac{p(D \mid f)\,p(f)}{\sum p(D \mid f)\,p(f)} \qquad \text{Eq. 3}$$

Where $p(f)$ is the prior probability distribution for *f* and $p(D|f)$ is the likelihood function for the data *D* given the value of *f*. In the absence of prior information it can be assumed that $p(f)$ is a uniform distribution from 0 to 1. Use can be made of expert judgment and experience in order to limit the lower and the upper bounds for the defect probability *f*. The likelihood function can calculated using the binomial distribution:

$$p(D \mid f) = \binom{n}{n_d}(1-f)^{n_d}\, f^{\,n-n_d} \qquad \text{Eq. 4}$$

Figure 2 illustrates the prior information on the distance between the shear reinforcement together with updated distribution based on the test results that verify the design value. It can be observed that the consideration of the test data focuses more narrowly the probability distribution around the design value.

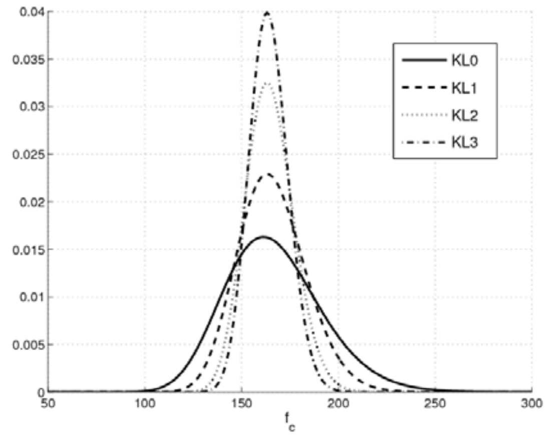


Figure 1. The prior and the updated probability distributions for concrete for different knowledge levels.

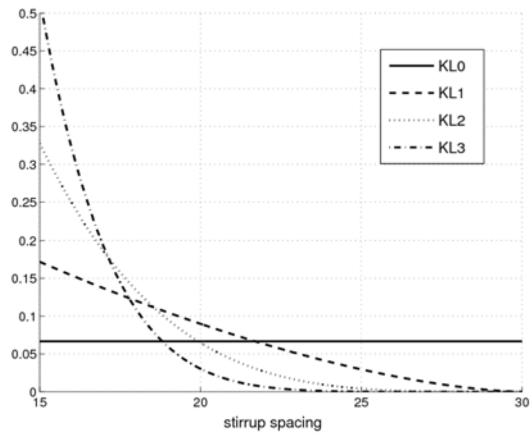


Figure 2. The uniform prior and the updated probability distribution for the distance between shear reinforcement.

2.3 *Calculating Structural Reliability using an Approximate Semi-Deterministic Method*

The confidence factors specified by Euro Code 8 are applied to the mean material properties. Obviously, the approach based on the application of the CF does not take into account explicitly the uncertainties. It would be interesting to investigate what would the application of CF achieve in terms of the seismic performance of the structure.

Consistent with the definition of the KL in the code, one can define the KL_o or the knowledge level before performing the tests. Therefore, for a structure in KL_o the application of the confidence factor implies utilizing smaller material property values. Figure 3 illustrates different values of the material property in question for different values of the CF. It can be observed from Figure 3 that increasing the confidence factor decreases the percentage of values smaller than the nominal value or in other words increases the *confidence* in the nominal value.

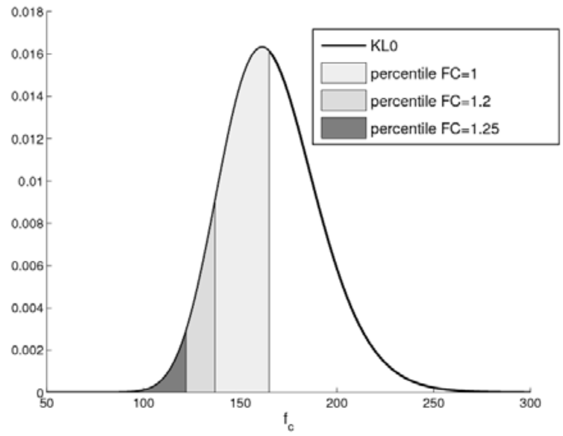


Figure 3 The percentiles corresponding to each CF on the prior probability distribution for concrete.

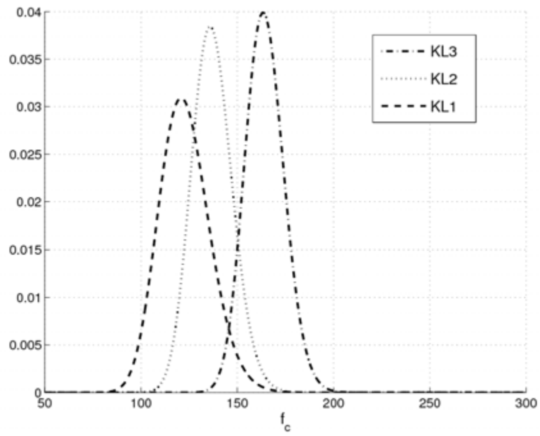


Figure 4. The code-based updated probability distributions for concrete for each knowledge level.

Figure 4 illustrates the probability distributions updated for each knowledge level where the mean value for resistance is divided by the corresponding confidence factor. It can be seen that with the increasing knowledge level, the standard deviation in the probability distributions decreases.

In order to map the above discussion into the global performance of the structure, it is assumed for simplicity that the percentiles in the material properties map out invariantly into the structural performance parameter. This approximation would have been exact if the non-linear structural analysis was *a strictly monotonic function*. Figure illustrates the structural fragility curve –

built deterministically- by calculating the structural performance variable for different values of CF and plotting them versus their corresponding confidence. It can be seen from figure that with increasing the CF, the structural performance parameter increases and exceeds unity for CF > 1. Therefore, the $Y_{CF>1}$ value corresponds to a Y value with a higher confidence compared to $Y_{FC=1}$. It should be noted that for KL levels higher than KL_0, the CF will map out to even higher confidences.

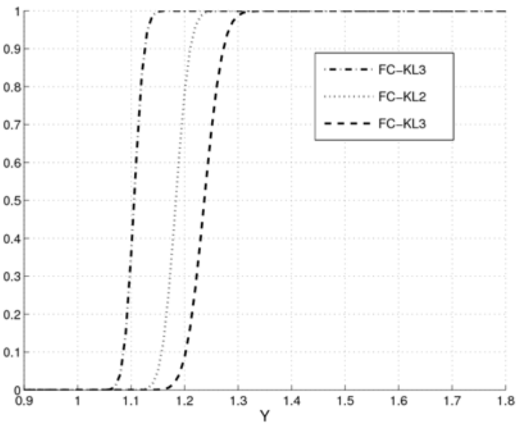


Figure 5. The (approximate) fragility curves corresponding to each CF.

2.4 *An efficient method for estimation of robust reliability*

The probability of failure given the set of parameters β (e.g., median and standard deviation of the fragility curve) is denoted by $P(F|\beta)$, the expected value (or the robust estimate) for the probability of failure given a set of values Y for the structural performance index can be expressed as [Jalayer et al., 2009]:

$$E[P(F \mid D] = \int_{\Omega} P(F \mid \beta) p(\beta \mid D) d\beta \qquad \text{Eq. 5}$$

where $p(\beta |D)$ is the posterior probability distribution for the set of parameters β given the data D and Ω is the space of possible values for β. In a similar way, the robust variance for the probability of failure can be calculated as:

$$\sigma_{P(F|D)} = E[P(F \mid D)^2] - E[P(F \mid D)]^2 \qquad \text{Eq. 6}$$

The structural reliability or the probability of failure in the case of a structure with modeling uncertainties (no uncertainty in the ground motion) can be expressed by a LogNormal cumulative distribution function (CDF) as following:

$$P(Y(\underline{\theta}) > y) = 1 - \Phi\left(\frac{\log \frac{y}{\eta_Y}}{\beta_Y}\right) \qquad \text{Eq. 7}$$

Where Y is the structural performance index and η_Y and β_Y are the median and the standard deviation (of the logarithm) for the probability distribution of the structural performance index. Using Bayesian inference, the posterior probability distribution for median and standard deviation based on data Y can be written as [Box and Tiao, 1999]:

$$P(\eta_Y, \beta_Y \mid Y) = k\beta_Y^{-(n+1)} \exp\left(-\frac{\nu s^2 + n(\log\eta_Y - \overline{\log Y})^2}{2\beta^2_Y}\right)$$

$$k = \sqrt{\frac{n}{2\pi}}\,\Gamma(\nu/2)^{-1}\left(\frac{\nu s^2}{2}\right)^{\nu/2} \qquad \text{Eq. 8}$$

where $Y=\{Y_1, \ldots, Y_n\}$ is the vector of n different realizations of the structural performance index, $\nu=n-1$, logY (overbar) is the mean value for logY and ns^2 is sum of the squares of the deviations from the mean value. The expected value and the standard deviation for the probability of failure can be calculated from Equations 5 and 6 based on the posterior probability distribution $p(\eta_Y, \beta_Y|Y)$ in Equation 8. Otherwise, the best-estimate values for the median and standard deviation can be calculated either as the maximum likelihood pair for the posterior probability distribution function or based on a given (e.g., 84%) confidence.

The structural reliability in the presence of modeling uncertainties and uncertainties in the representation of the ground motion can be calculated from the following LogNormal CDF:

$$P(Y(\underline{\theta}) > y \mid S_a) = 1 - \Phi\left(\frac{\log y - \log\eta_{Y|S_a}}{\beta_{UT}}\right) \qquad \text{Eq. 9}$$

$$\beta^2_{UT} = \beta^2_{Y|S_a} + \beta^2_{UC}$$

where $\eta_{Y|Sa}$ is the median for the probability distribution of the structural performance index and β_{UT} is the standard deviation for the probability distribution of the structural performance index. The terms $\beta_{Y|Sa}$ and β_{UC} represent the effect of the uncertainty in the ground motion representation, the uncertainty in the material properties and the structural details, respectively. It should be noted that Equation 9 yields the structural fragility; after integrating it with the hazard function for the spectral acceleration, the hazard function for the structural performance variable Y can be obtained.

Suppose that a selection of n ground motion records are used to represent the effect of ground motion uncertainty on the structural performance index. Let $S_{a,i}$ and Y_i represent the spectral acceleration and the performance index for the ground motion record i, respectively. The posterior probability distribution for standard deviation can be calculated as:

$$P(\beta_{UT} \mid Y) = \Gamma(\nu/2)^{-1}\left(\frac{\nu s^2}{2}\right)^{\nu/2} \beta_{UT}{}^{-(\nu+1)} \exp\left(\frac{-\nu s^2}{2\beta^2{}_{UT}}\right)$$

Eq. 10

The data pairs (Y, Sa) are gathered by calculating the structural performance measure for the set of n ground motion records applied at the structural model generated by different realizations of material mechanical properties and structural detailing parameters. νs^2 is equal to the sum of the square of the errors for a linear regression of $\log Y$ on $\log S_a$ and a and b are the regression coefficients. The joint posterior probability distribution for the coefficients of the linear regression $\theta = (\log a, b)$ can be calculated as:

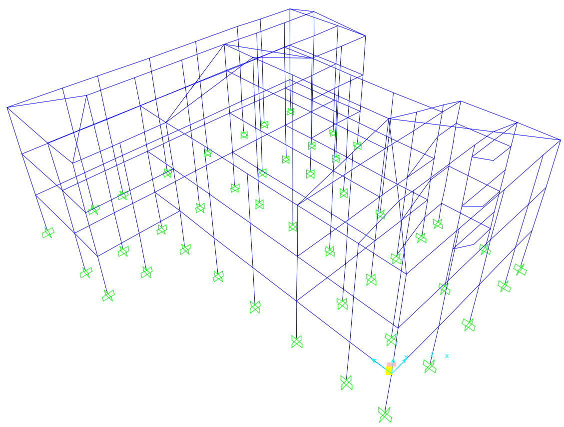
$$P(\theta \mid Y, S_a) = k\left[1+\frac{(\theta-\hat{\theta})^T X^T X(\theta-\hat{\theta})}{\nu s^2}\right]^{-\frac{\nu+1}{2}}$$ Eq. 11

$$k = \frac{\Gamma\left(\frac{n}{2}\right)\sqrt{n\sum \log S^2{}_{a,i} - (\sum \log S_{a,i})^2}}{\nu s^2 \Gamma\left(\frac{1}{2}\right)^2 \Gamma\left(\frac{n}{2}-1\right)}$$

which is a *bivariate t-distribution* where X is a nx2 matrix whose first column is a vector of ones and its second column is the vector of log $S_{a,i}$ and θ is the 2x1 vector of regression coefficients log a and b. The median and the standard deviation for the probability distribution for $Y|S_a$ can be taken equal to the maximum likelihood estimates $\eta_Y = aS_a{}^b$ and $\beta_{Y|Sa}=s$. The robust estimates for the expected value and the standard deviation of the failure probability can be obtained from Equations 5 and 6 based on the product of the posterior probability distributions $p(\theta|Y,S_a)$ and $p(\beta_{UT}|Y,S_a)$ in Equations 10 and 11, assuming they are independent.

3 NUMERICAL EXAMPLE

As the case-study, an existing school structure located in Avellino, Italy is considered herein. The structure is situated in seismic zone II according to the Italian seismic guidelines OPCM. The structure consists of three stories and a semi-embedded story and its foundation lies on soil type B. For the structure in question, the original design notes and graphics have been gathered. The building is constructed in the 1960's and it is designed for gravity loads only, as it is frequently encountered in the post second world war construction.



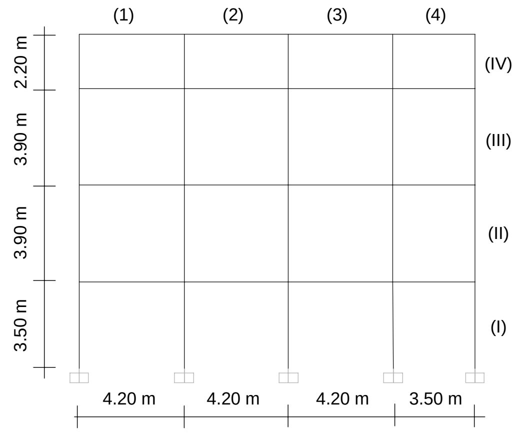


Figure 5(a) The tri-dimensional view of the scholastic building 5(b) The central frame of the case-study building

In Figure 5a, the tri-dimensional view of the structure is illustrated; it can be observed that the building is highly irregular both in plane and elevation. In order to reduce the computational effort, the main central frame in the structure is extracted and used as the structural model (Figure 5b). The columns have rectangular section with the following dimensions: first storey:

40x55cm^2, second storey 40x45cm^2, third storey: 40x40cm^2, and forth storey: 30x40 cm^2. The beams, also with rectangular section, have the following dimensions: 40x70cm^2 at first and second storey, and 30x50 cm^2 for the ultimate two floors. It can be inferred from the original design notes that the steel re-bar is of the type Aq40 and the concrete has a minimum resistance equal to 180 kg/ cm^2 [DL1939]. The finite element model of the frame is constructed assuming that the non-linear behavior in the structure is concentrated in plastic hinges.

3.1 The structural performance index

When only the structural modeling uncertainties are considered, the definition of structural capacity in this work is based on the limit state of severe damage as proposed by the Italian Code. That is, the onset of critical behavior in the first element, characterized by member chord rotations larger than the corresponding ultimate chord rotation capacity. The structural demand is characterized by the intersection of the code-based inelastic design spectrum and the static pushover curve transformed into that of the equivalent SDOF system. As an index for the global structural performance, the ratio of structural demand to capacity is used. The component shear failure demand to capacity ratios are also considered; they are combined with the CSM demand to capacity using the *cut set theory* (see below).

When the ground motion uncertainty together with the modeling uncertainties are taken into account, the structural performance index is characterized based on the concept of *cut-sets* in structural reliability. A structural cut-set is defined as a set of structural components that, once all of them have failed, they can transform the whole structure or part of it into a mechanism. Among the set of all possible cut-sets, the critical cut-set is the one that first forms a global mechanism. Therefore, the performance index is taken as the demand to capacity ratio of the strongest component of the weakest cut-set. In the current work, three types of global mechanism are considered: (a) ultimate rotation capacity in the columns (b) formation of soft stories (c) shear failure in the columns. The component yield rotation, ultimate rotation and shear capacities are calculated according to the new Italian Unified Code (MIN.LL.PP 2008a,b). It should be noted that the structural performance in both cases signals failure when it is great than unity and signals no structural failure when it is less than or equal to unity.

3.2 Calculating the structural Fragility: CSM

The structural fragility based on the capacity spectrum method is estimated employing the efficient Bayesian method described above based on the structural performance parameter for a set of 20 Monte Carlo (MC) realizations of the structural model. These realizations take into account the uncertainties in the material properties and the structural defects. The probability distributions for the uncertain parameters are updated according to the increasing knowledge levels KL_o, KL_1, KL_2 and KL_3. As stated before, these knowledge levels are achieved based on the EC8 specifications tabulated in Table 1. Thus, for each knowledge level, the 20 realizations of the structural model are generated from the (updated) probability distributions corresponding to the KL's and based on the results of tests and inspections. Since the results of tests and inspections available for the structure in question did not exactly match the EC8 criteria, the test results used herein are simulated assuming that all the inspections performed verify the original design values. Figure 6 demonstrates the robust fragility curves (the probability of failure for a given value of *Y*) obtained using Equations 5, 7 and 8 for knowledge levels KL_1, KL_2 and KL_3. The fragility curves in grey for each knowledge level illustrate the corresponding robust fragility plus its standard deviation.

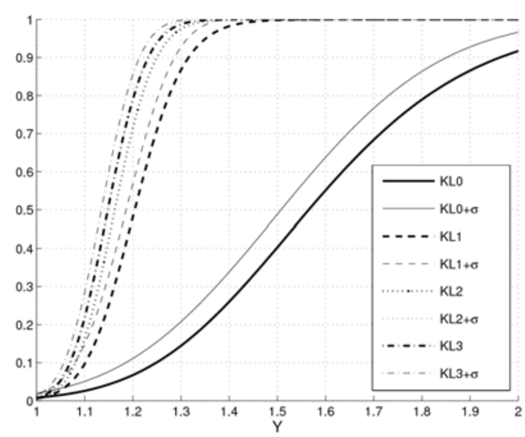


Figure 6 The structural fragility curves for the knowledge levels KL_o, KL_1, KL_2 and KL_3.

It can be observed that the upon increasing knowledge levels the both the median and the dispersion in the fragility curves (β_Y and η_Y in Equation 1) decrease as the test results all verify the nominal values. However, it can be seen that the structure does not verify the SAF-FEMA criteria in Equation 1 in none of the knowledge levels. That is, because the median η_Y is already greater than unity. Figure 7 shows the robust fragility curves for each knowledge level together with the approximate fragility curves developed based on the code-specified method plotted in grey. It can be observed that while the code-based fragility curves remain distinct and with the same dispersion, the robust fragility curves get closer and have smaller dispersion as the knowledge levels rises.

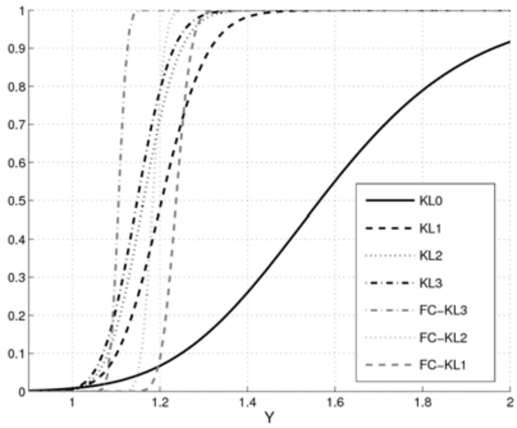


Figure 7 The structural fragility curves for the knowledge levels KL_o, KL_1, KL_2 and KL_3.

3.3 An Approximate Method for calculating Structural reliability: Dynamic Analyses

It is shown previously in this work how the CF can be viewed in an approximate way from the stand-point of structural reliability using the non-linear static analyses. In a similar way, it can be shown how the CF can be viewed in the dynamic case. A set of 7 records are chosen compatible with the code-specified spectrum [EC8]. For each CF specified in the code, the structural performance variable for the set of records is calculated for a structural model (without defects) with material properties divided by that CF. The structural performance variable is related to the spectral acceleration using linear regression with parameters $\eta_{Y|Sa}$ and $\beta_{Y|Sa}$. The structural fragility is calculated from Equation 9 setting β_{UC} equal to zero. Finally, the structural fragility is integrated with the spectral acceleration hazard curve (extracted from the site of INGV) in order to calculate the probability of failure. The resulting hazard curves corresponding to different values of CF are plotted in Figure 8. It should be noted that dispersion in these hazard curves reflects only the record-to-record variability. In a way, similar to Figure 5 for the static case, using increasing values of CF is equivalent to taking into account the structural modeling uncertainties by taking hazard curves (including only the ground motion uncertainty) corresponding to higher confidence levels.

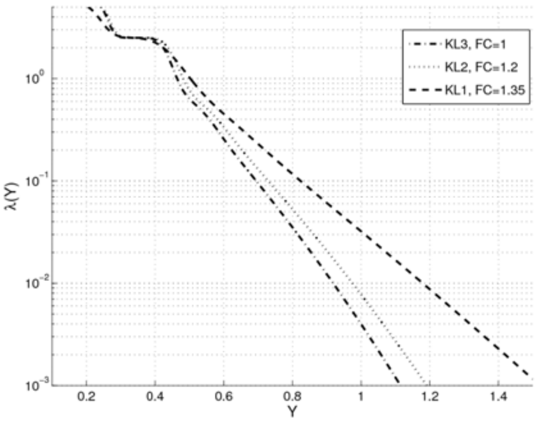


Figure 8 The approximate code-based hazard curves for the knowledge levels KL_o, KL_1, KL_2 and KL_3.

3.4 Calculating the Structural Reliability: The Dynamic Method

The structural hazard curve for increasing levels of knowledge is calculated in this stage by integrating the robust fragilities and the spectral acceleration hazard curve at the site of the structure. For each level of knowledge, the robust fragility is calculated from Equations 5, 9, 10 and 11 using a set of 30 MC realization of the structural model. The set of MC realizations for each KL are generated based on the corresponding (updated) probability distributions.

The resulting hazard curves are plotted in Figure 9. The grey hazard curves correspond to robust fragility plus one standard deviation for each knowledge level. It can be observed that with increasing the knowledge level KL, the mean annual frequency of exceeding the structural performance parameter Y decreases. It can be shown (Jalayer and Cornell 2008) that calculating the left-hand side of Equation 2 for a given acceptable probability P_o is equivalent to finding the value corresponding to P_o from the hazard curve for structural performance parameter. For example for an acceptable probability of P_o=0.002 or 10% in 50 years, the structure does not verify for none of the KL.

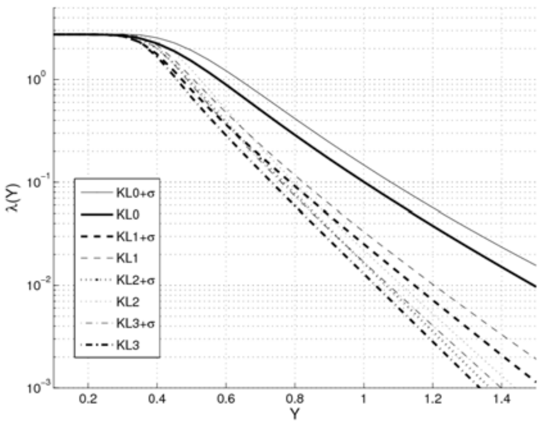


Figure 9 The hazard curves for robust fragility and robust fragility plus standard deviation for knowledge levels KL_o, KL_1, KL_2 and KL_3.

Figure 10 illustrates the hazard curves based on the robust fragilities plotted together with the approximate code-based hazard curves.

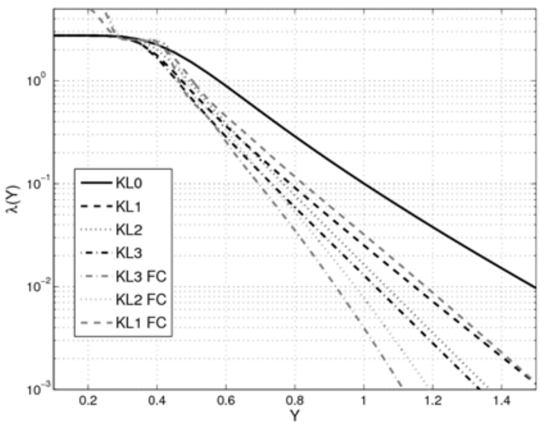


Figure 10 The robust hazard curves and the approximate code-based hazard curves for the knowledge levels KL_o, KL_1, KL_2 and KL_3.

4 SOME PERSPECTIVES FOR EC8 IN LIGHT OF THE ITALIAN EXPERIENCE

The knowledge levels (KL) defined by the code leave a lot of room for interpretation. In other words, the code-based definition for KL does not lead to a unique configuration of tests and inspections. Moreover, it is not clear what level of structural reliability does the application of the confidence factors guarantee. Hence, with the emerging of performance-based design and life-cycle cost analysis in earthquake engineering, there seems to be a need for a code-based method that bridges the different knowledge levels to structural reliability and probabilistic structural performance assessment. A proper evaluation of the structural performance needs to take into account directly the uncertainties in the structural modeling parameters. Thus, the suitable approach for assessment of existing buildings is the probabilistic one which accounts for all the uncertainties. In this sense, the approach of CF can be seen as a deterministic way of dealing with a probabilistic problem.

Intuitively speaking, the relation between the confidence factors and the knowledge levels seems to be highly dependent on the results of in-situ tests and inspections. Therefore, it is necessary to adopt a general probabilistic framework for updating the probability distribution for the uncertain parameters based on the test results. The Bayesian framework for inference seems to be perfectly suitable for this end; as it can sequentially incorporate the incoming tests and inspection results without discarding any prior information available.

There seems to be a need for simple and approachable probabilistic performance-based alternatives to the CF method. These methods can be incorporated in increasing levels of sophistication depending on the importance of building under assessment. The simplified safety-checking format adopted by the American SAC/FEMA guidelines for the assessment and retrofit of existing buildings seems to be an interesting example. This simplified method takes into account the effect of all sources of uncertainty (GN, structural modeling) in the global performance of the structure. This format is expressed as a function the statistical parameters of the structural performance parameter.

In the context of a simple performance-based assessment approach, different classes of existing buildings can be identified and analyzed. The prior probability distributions for the structural modeling uncertainties can be classified and tabulated based on the surveys of expert opinion and experience. It is important to identify those uncertain parameters that affect the structural response in a dominant way (e.g., the material properties, the stirrup spacing). These prior probability distributions are going to be updated based on the results of tests and inspections. The updated probability distributions are constructed for various KL's, based on special cases of tests and inspection results. Finally, for different classes of structures and different levels of knowledge (and a few special cases of inspection results), the best-estimate values for the parameters defining the adopted safety-checking format/structural fragility can be tabulated. In the case of strategic buildings, it would be useful to recommend some relatively simple and approachable methods suitable for case-specific estimation of the parameters defining the safety checking format and/or structural fragility. This work is a preliminary effort in classifying (for different levels of analysis sophistication) different methods suitable for the performance-based assessment of existing buildings.

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